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Research Paper

The influence of soil disturbance on material properties and micro-structure of cement-treated soil

M. Makino [1], T. Takeyama [2] and M. Kitazume [3]

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ABSTRACT

The Pneumatic Flow Mixing Method was developed in Japan, in which dredged soft soil is mixed with small amount of chemical binder, usually cement, in a pipe during transporting by compressed air. In some applications, the stabilized soil is temporary placed at provisional space, and is excavated and reclaimed at the final site due to several reasons. In the cases, the stabilized soil in which the chemical reaction has already proceeded is disturbed during the excavation and transportation processes, which causes considerable decrease in the strength at the final site. It is important to investigate the soil disturbance effect for its further applications. The authors have started a research project to investigate the soil disturbance effect on the mechanical properties of cement stabilized soil. The unconfined compression tests were performed on three types of laboratory mixed cement stabilized Kaolin clay, stabilized soil without disturbance and disturbed after 3 or 7 days curing. The study revealed that the soil disturbance influenced the stress - stain behavior of the soils considerably. In this paper, a part of the on-going research results is presented as well as the soil preparation and test procedure.

1. Introduction

A huge amount of soft soil is dredged at many ports and rivers every year to maintain enough sea route and sea berth or to preserve their environment. These soft soils are used to be dumped at disposal sites constructed at coastal area. Recently it becomes quite difficult to construct the disposal site for dredged soil and subsoil, because of restriction of environmental protection and economic reason. As the dredged soil usually has a high water content of the order of several hundred percent, reclaimed land constructed with them is so soft and high compressible that no superstructure can be constructed on it without any soil improvement. Vertical drain method is one of the most frequently used soil improvement techniques to improve such a clayey deposit. However the method requires relatively long period to complete the consolidation.

These circumstances promote to use dredged soft soil as a reclamation material, and many studies and researches have been carried out to investigate stabilizing methods for the soil and to establish recycling system for soft soil (Correia *et al.*, 2013, EuroSoilStab, 2001, Japan Cement Association,2012). The Pneumatic Flow Mixing Method was developed for beneficial use of dredged soil in Japan, in which dredged soft soil is mixed with small amount of chemical binder, usually cement, in a pipe during transporting by compressed air and is

[1] Former graduate student, Department of Civil Engineering, Tokyo Institute of Technology, Tokyo 152-8552, JAPAN, makino.m.ab@gmail.com
[2] Associate professor, Department of Civil Engineering, Kobe University, Kobe 657-8501, JAPAN, takeyama@people.kobe-u.ac.jp
[3] Corresponding author, Professor, Department of Civil Engineering, Tokyo Institute of Technology, Tokyo 152-8552, JAPAN, kitazume@cv.titech.ac.jp
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placed for land reclamation (CDIT 2001, Kitazume and Satoh 2003). The characteristics of the stabilized soil have been studied by laboratory and field tests (Hayano and Kitazume 2005). In some applications, the stabilized soil is temporary placed at provisional space, and is excavated and reclaimed at the final site after certain period due to several reasons. In the cases, the stabilized soil in which the chemical reaction has already proceeded is disturbed during the excavation and transportation processes, which causes considerable decrease in the stabilized soil strength at the final site (Kitazume *et al*. 2007). The physical and mechanical properties of stabilized soil have been investigated in detail for many years (e.g. Saitoh 1988, Terashi *et al*. 1977, 1980 and 1983). It is important to investigate the effect of soil disturbance on the physical and mechanical properties of the stabilized soil for promoting further applications of the method.

The authors have conducted a research project to investigate the effect where series of laboratory tests are properties of the cement stabilized Kaolin clay in the unconfined compression test.

2. Laboratory tests

2.1 Material and test procedure

The Kaolin clay was stabilized and tested in unconfined compression, with Ordinary Portland Cement (OPC) as a binder. The physical properties of the Kaolin clay are summarized in Table 1. The powder of Kaolin clay was throughout mixed with tap water to make uniform cement slurry having the water content of 120%. The cement slurry and the OPC in dry form, *aw* = 5% and 10%, were mixed for 5 min., where the binder content, *aw*, is defined as the dry weight ratio of binder and soil. After the mixing, the stabilized soil was molded by the tapping technique to remove any trapped air according to the JGS Standard (2009). In this technique, the stabilized soil mixture was put into a plastic mold, 50 mm in diameter and 100 mm in height, in three layers. For each layer, the mold was tapped against floor 100 times. After filling, the top surface of the soil mixture were smoothed out and sealed with vinyl sheet for curing.

In the case of disturbed soil specimen, the stabilized soil mixture was stored and cured in an airtight plastic bag first to avoid any change in water content. After 3 or 7 days, the soil mixture in the bag was through disturbed by being mixed for 2 min. in a mixer and compacted into the mold in the same way as the un-disturbed soils.

After the prescribed curing period, the soil samples were subjected to the unconfined compression test. Table 2 shows the characteristics of the specimens. As the triaxial test apparatus was used in this study, the axial strain ratio of 0.4 %/min. was adopted for the test. The hand vane share test was conducted on the stabilized soil immediately after the disturbance to measure their undrained shear strength. The vane test was also performed on some disturbed stabilized soils whose strength was not large enough for the unconfined compression test. The two types of vane apparatus were used, whose diameter and height were 10 mm and 20 mm, or 20 mm and 40 mm respectively. The hand cone penetration test was also performed on several soil specimens in order to calibrate the undrained shear strength measured in the unconfined compression test, the hand vane shear test and the cone penetration test. The cone apparatus was a steel column, whose diameter and top apex were 20 mm and 30 degree respectively. In the test, the penetration load was measured at the penetration depths of 10 mm, 20 mm and 37.3 mm (when the tapered section was fully penetrated). The test was performed five times per case to obtain reliable measured data.

Table 1. Physical properties of Kaolin clay.

Property	Value
Soil particle density, ρ_s (g/cm^3)	2.61
Liquid limit, w_l (%)	77.5
Plastic limit, w_p (%)	30.3
Plasticity index, I_p (%)	47.2

Table 2. Specimen numbers and characteristics of specimens.

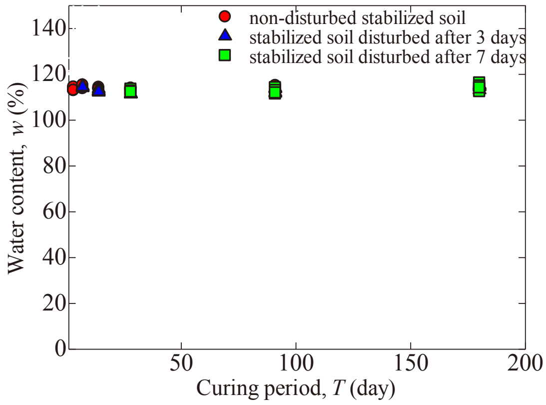
Specimen No.	*aw*	Disturbance
SS_5N	5%	un-disturbed
SS_5D3	5%	disturbed after 3 days
SS_5D7	5%	disturbed after 7 days
SS_10N	10%	un-disturbed
SS_10D3	10%	disturbed after 3 days
SS_10D7	10%	disturbed after 7 days

3. Test results and discussions

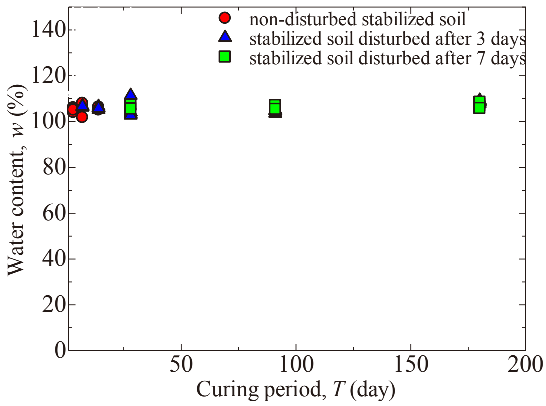
As the research is on-going, the test results up to 6 months curing are shown in this study.

3.1 Water content

The water content of the specimen is shown in Figs. 1(a) and 1(b) for *aw* = 5% and 10% respectively. The figures show that the water contents of three types of samples are almost coincide and show negligible change throughout the curing period up to 6 months. This reveals that the soil disturbance process was conducted in undrained condition and the curing process has been well controlled.



(a) *aw* = 5%



(b) *aw* = 10%

Fig. 1. Water content with curing period.

3.2 Unconfined compression test

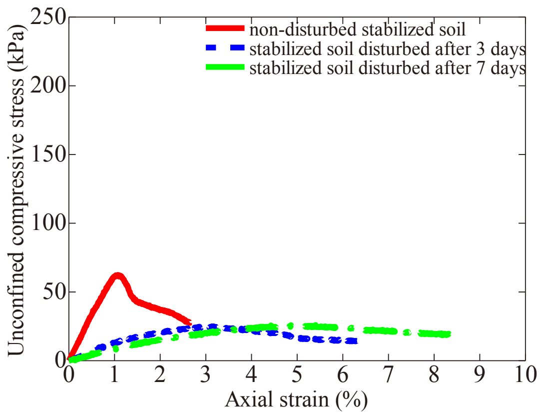
3.2.1 Stress-strain curve

The stress-strain curves of un-disturbed stabilized soil, stabilized soils disturbed after 3 days and 7 days at curing period of 28 days are shown in Figs. 2(a) and 2(b) for *aw* = 5% and 10% respectively.

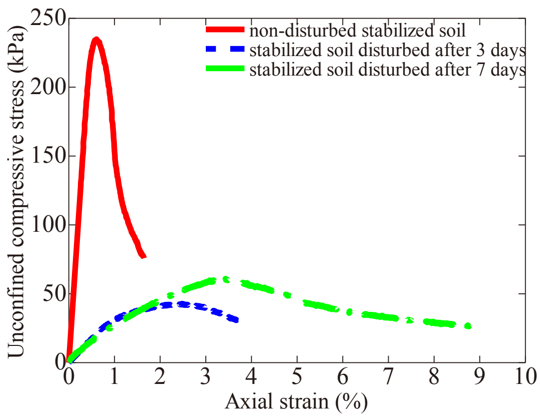
As seen in Fig. 2(a), in the case of *aw* = 5%, the deviator stress of the un-disturbed stabilized soil, SS_5N (the specimen numbers and the characteristics of specimens are summarized in Table 2), increases rapidly with the axial strain increases, reaching peak strength when the axial strain is approximately 1% and then decreasing quickly after the peak strength. The deviator stress of the stabilized soil disturbed after 3 days, SS_5D3, increases gently as the axial strain increases, reaches a peak when the axial strain is approximately 3% and decreases gently. Its peak strength is smaller than that of SS_5N. The deviator stress of the stabilized soil disturbed after 7 days, SS_5D7, increases and decreases similarly to SS_5D3 and the peak strength is almost the same but at a large axial strain of approximately 5%.

Figure 2(b) shows the measured data in the case of *aw* = 10%, while the unconfined compressive strengths of three types of the stabilized soils are larger than those of *aw* = 5%. In particular, the unconfined compressive strength of the un-disturbed stabilized soils, SS_10N, is quite larger than that of SS_5N. The axial strain at the peak strength in SS_10N is smaller than that of SS_5N.

The figures reveal that the un-disturbed stabilized soil shows a brittle characteristic with quite large strength at small axial strain and quite small residual strength but the disturbed stabilized soils show a ductile characteristic with small strength and stiffness.



(a) *aw* = 5%



(b) *aw* = 10%

Fig. 2. Stress-strain curves at the curing period of 28 days.

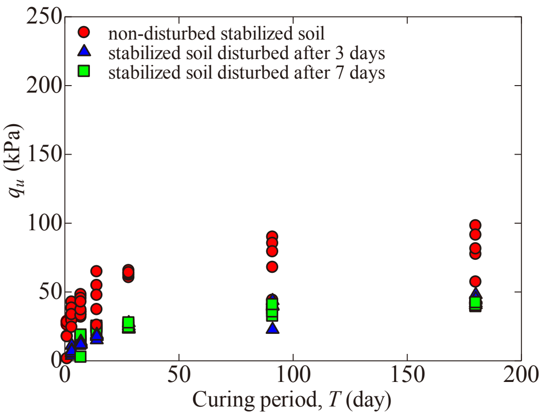
3.2.2 Unconfined compression strength

The relationship between the unconfined compressive strength, q_u, and the curing period is shown in Fig. 3. The hand vane shear test and hand cone penetration test were performed on some disturbed stabilized soils whose strength was not large enough for the unconfined compression test. The vane shear strength and cone penetration resistance were converted to the unconfined compressive strength based on the relationships obtained in advance and shown in Figs. 3(a) and 3(b) for *aw* = 5% and *aw* = 10% respectively.

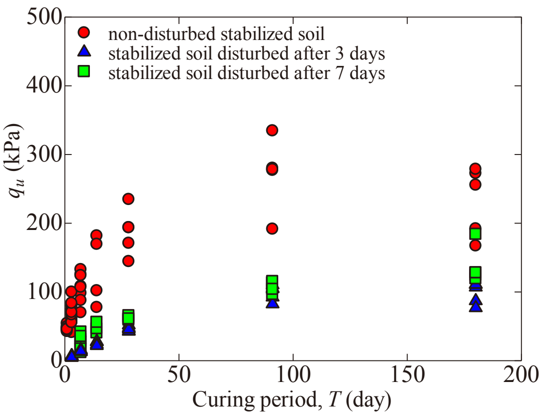
As shown in Fig. 3(a), the unconfined compressive strength, q_u, of SS_5N increases to 70 kPa with the curing period. The q_u of SS_5D3 and SS_5D7 are smaller than that of SS_5N. After being disturbed, the strengths of these disturbed stabilized soils increase gently with the curing period. It can be seen in the figure that the strengths of the disturbed stabilized soil are almost same irrespective of the age of disturbance.

In Fig. 3(b), the q_u of SS_10N increases rapidly during the early curing period, and then increases gradually with the curing period. The strengths of SS_10D3 and SS_10D7 also increase with the curing period but the increasing ratio is smaller than that of SS_10N. The magnitudes of q_u of SS_10D3 and SS_10D7 are smaller than SS_10N. The strength increase phenomenon of the disturbed stabilized soils are almost the same as those of *aw* = 5% (Fig. 3(a)). Though it can be seen that the q_u of SS_10D7 is somewhat larger than that of SS_10D3, further study may be necessary to study the reason.

In order to clarify the effect of the soil disturbance on the strength, the strength ratios are shown in Figs. 4(a) and 4(b) for *aw* = 5% and 10% respectively, where the



(a) *aw* = 5%

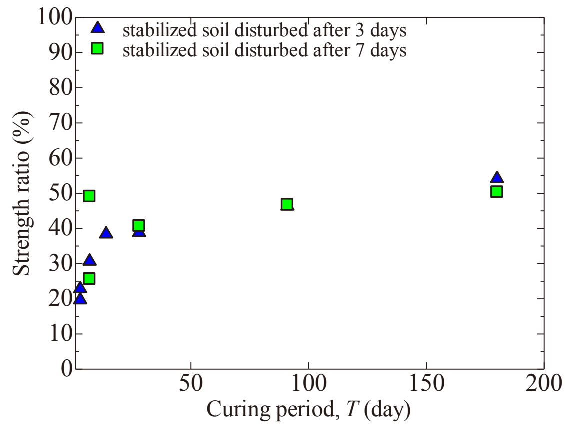


(b) *aw* = 10%

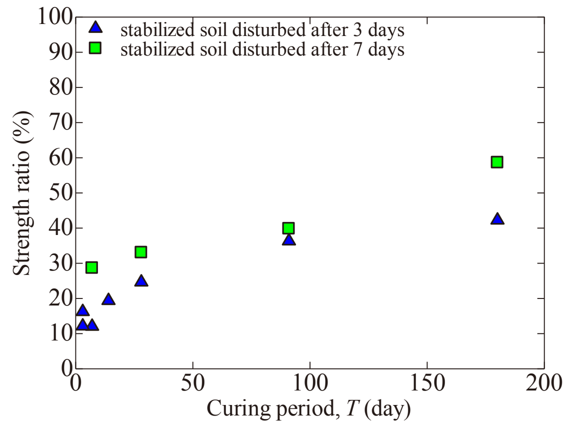
Fig. 3. Relationship between unconfined compressive strength and curing period.

strength ratio is defined by the unconfined compressive strength of the disturbed clay against that of un-disturbed clay. In Fig. 4(a), *aw* = 5%, it is found that the strength ratio of SS_5D3 is about 20% soon after the disturbance and gradually increases to about 40% with the curing period. The strength ratio of SS_5D7 shows almost same strength ratio as that of 3 days.

In Fig. 4(b), *aw* = 10%, it is shown that the strength ratio of SS_10D3 is about 10 to 20% soon after the disturbance and increases to about 40% with the curing period. The strength ratio of SS_10D7 is about 30% soon after the disturbance which is comparatively larger than that of SS_10D3. The ratio increases with the curing period without any a constant ratio. The figures reveal that the soil disturbance causes large strength reduction soon after the disturbance and the strength ratio then gradually increases. The effect of the soil disturbance is more dominant in the stabilized soil with large binder content than that with small binder content.



(a) *aw* = 5%



(b) *aw* = 10%

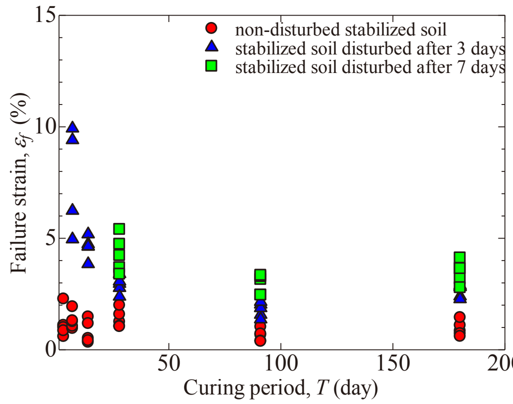
Fig. 4. Relationship between unconfined compressive strength ratio and curing period.

3.2.3 Axial strain at failure

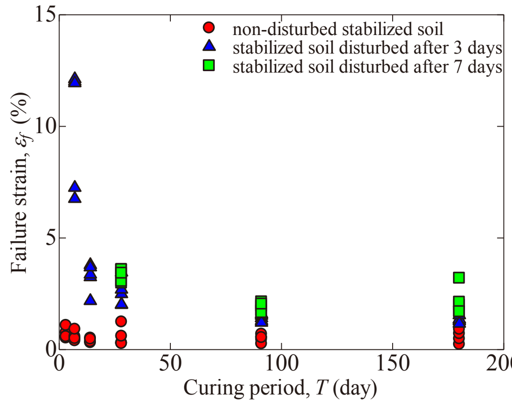
The relationship between the axial strain at failure, ε_f and the curing period is shown in Figs. 5(a) and 5(b) for *aw* = 5% and 10% respectively. In Fig. 5(a), *aw* = 5%, the

ε_f of SS_5N is around 1 to 2% and remains constant regardless of the curing period. In the case of SS_5D3, the hand vane test and cone penetration test were performed at the 3 days' curing instead of the unconfined compression test as explained before. Therefore no data in the axial strain was measured in these tests. The ε_f of SS_5D3 at 7 days' curing (4 days after the disturbance) is quite large of about 5 to 10% and decreases rapidly to about 3% with the further curing. In the case of SS_5D7, no data was measured before 28 days' curing due to the above reason. At 28 days' curing (21 days after the disturbance), it is found that the ε_f is about 3 to 5%, which is larger than that of SS_5D3. Although its phenomenon in the ε_f was not measured before 28 days, it can be assumed that the ε_f is quite large value at the disturbance and decreases with the curing period, as similar to that of SS_5D3.

In Fig. 5(b), *aw* = 10%, the axial strain at failure of SS_10N is around 1% and remains constant regardless of the curing period. In the case of the disturbed stabilized soils, a similar phenomenon as explained above can be seen in cases of SS_10D3 and SS_10D7.



(a) *aw* = 5%

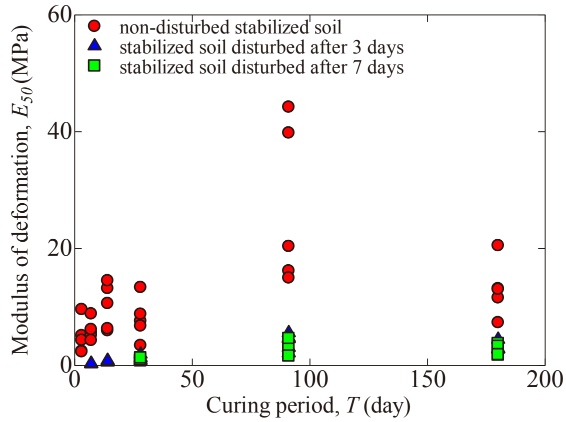


(b) *aw* = 10%

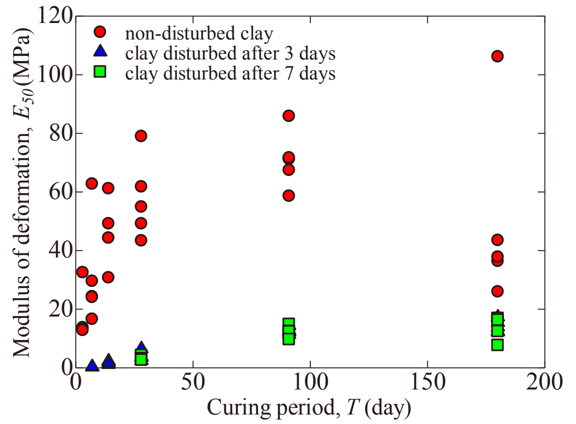
Fig. 5. Relationship between axial strain at failure and curing period.

3.2.4 Elastic modulus

The relationship between the elastic modulus, E_{50}, and the curing period is shown in Figs. 6(a) and 6(b) for *aw* = 5% and 10% respectively. In Fig. 6(a), *aw* = 5%, the E50 of SS_5N slightly increases with the curing period. In the case of the disturbed soils, SS_5D3 and SS_5D7, on the other hand, the E_{50} is considerably smaller than that of SS_5N and remains almost constant with the curing period. As mentioned before, the E_{50} was not measured on some disturbed soils. In Fig. 6(b), *aw* = 10%, the E_{50} of SS_10N is quite large value of 10 to 30 MPa initially and increases rapidly to 40 to 80 MPa with the curing period. In the case of the disturbed soils, the E50 values are small and then increases only slightly with the curing period.



(a) *aw* = 5%



(b) *aw* = 10%

Fig. 6. Relationship between elastic modulus and curing period.

4. Microscopic examination

A microscopic view of the stabilized soils was examined to clarify the change in their mechanical behavior during curing. A microscope used in this study was HITACHI, S-3400N, as shown in Fig. 7, where microscopic views of stabilized soil were taken under the accelerating voltage of 15 kV and the illumination current of 50 μA condition. Five types of stabilized soil were prepared as shown in Table 3. All the soils were dried at 110 degree in Celsius in an oven, before taking picture.

Figure 8 shows the microscopic photographs of the stabilized soils. In general, dark and white parts, and completely black part in the photograph correspond to flat surface and edge of cement hydrates, and void, respectively.

In the specimen (1), Fig. 8(a), quite large number of cobb-like fine particles can be seen. It is estimated that high strength of the sample was mobilized by these fine particles together with closely tightened fine particles by the help of the hydrates adhesion.

In the specimen (2) as shown in Fig. 8(b), on the other hand, no fine particles can be found but quite large number of small voids can be found. These voids might act as sponge when the soil was subjected to loading. It is considered that these voids might make the specimen ductile behavior.

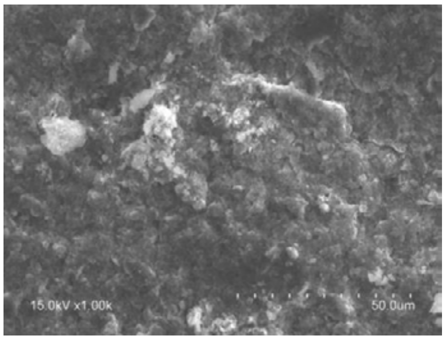
In the specimen (3), as shown in Fig. 8(c), no voids can be seen, but it is found that coarse particles are closely compacted.

In the specimen (4), Fig. 8(d), small voids can be seen in some places, together with relatively large void at the center which might not be caused by mechanical action but by dissolution of calcium ion in the long-term curing. This dissolution of calcium ion may cause the

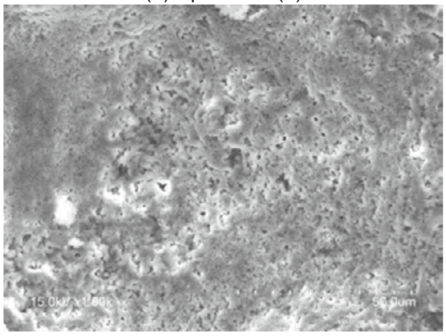
Fig. 7. HITACHI S-3400N microscope.

Table 3. Condition of specimen.

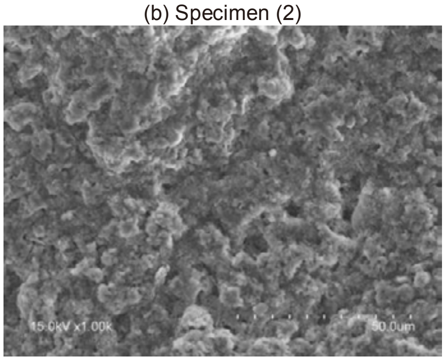
Specimen No.	Binder content, *aw*	Disturbance	Curing period
(1)	15%	un-disturbed	14 days
(2)	15%	disturbed at 3days	14 days
(3)	15%	disturbed at 3days	4 days
(4)	10%	un-disturbed	1.5 years
(5)	10%	disturbed at 3days	1.5 years



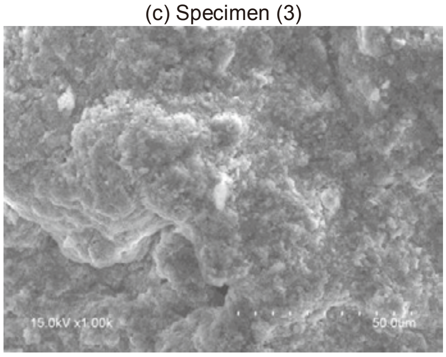
(a) Specimen (1)



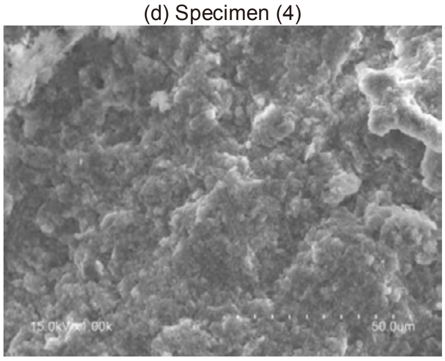
(b) Specimen (2)



(c) Specimen (3)



(d) Specimen (4)



(e) Specimen (5)

Fig. 8. Microscopic photos (*1000).

decrease in the strength.

In the specimen (5), Fig. 8(e), it can be seen that the coarse particles are formed without small voids which was found in the specimen (2).

According to the microscopic examinations, it is estimated that many fine voids are generated by the disturbance effect, which cause strength reduction. With curing period, these voids are gradually filled with hydrate, which gives rise to strength gain and brittle behavior.

5. Conclusions

The series of laboratory tests was performed to discuss the effect of soil disturbance on the water content and the stress - strain characteristics of the cement stabilized Kaolin clay in the unconfined compression test. As the research is on-going, the test results up to 6 months curing are shown in this paper. The major conclusions derived in the study are briefly summarized as follow.

(1) The un-disturbed stabilized soil shows brittle behavior with large peak strength and large elastic modulus and small axial strain at failure. The unconfined compressive strength of the un-disturbed stabilized soil increases with the curing period up to 6 months.

(2) The unconfined compressive strength of the stabilized soils decreases considerably to 10 to 20% of that of the un-disturbed stabilized soils due to the disturbance. The strength gradually increases to approximately 25 to 40% of that of the un-disturbed stabilized soils with the curing period. The failure strain of disturbed stabilized soils is also influenced by the disturbance. The failure strain of disturbed soils is found to be larger than that of the un-disturbed soils and the failure strain of disturbed soils decreases quickly with the curing period. The elastic modulus of the disturbed stabilized soils is considerably smaller than that of un-disturbed stabilized soils regardless of the binder content.

(3) According to the microscopic examinations, it is estimated that many fine voids are generated by the disturbance effect, which causes strength reduction. With a curing period, these voids are gradually filled with hydrate, which gives rise to strength gain and brittle behavior.

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Symbols and abbreviations

E_{50}	Elastic modulus
q_u	Unconfined compressive strength
ε_f	Axial strain at failure